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Structural Evaluation and Load Testing of Flack Bridge, Mitchell's Creek

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Abstract:

This report describes the structural evaluation and load test studies performed on Flack River Bridge. The Flack River Bridge consists of pony trusses spanning 21.3 m with a concrete deck cast over stringers and floor beams. The structural evaluation investigations showed that the deck and stringers have adequate capacity and the floor beams and truss top chords and bottom chords have less strength than required. Most of the web members had adequate strength. The load tests carried out on the structure showed that the floor system and the truss members behaved linear elastically under the applied truck loads. The test truck loads were designed to produce force effects reaching the ultimate load levels described in Ontario Highway Bridge Design Code (OHBDC). The tests confirm that the bridge has adequate strength as a system to sustain OHBDC loads without posting. It is recommended that repairs be carried out 1) to control the deck deterioration and to 2) improve the functioning of the bearings.

Key Words: Flack Bridge, pony truss bridge, bridge test, load testing

Comments:

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1/ INTRODUCTION

Flack Bridge is located in the County of Wellington on County Road 11 immediately south of Highway 9 crossing Mitchell's Creek. It is a steel pony truss bridge with an exposed concrete deck, built in 1954. McCormick, Rankin and Associates inspected and evaluated the bridge in July 1982. The exposed deck showed significant scaling and spalling. The bridge evaluation study indicated that the steel truss was under strength and required posting to carry OHBDC loads. The consultants, McCormick, Rankin and Associates, made a proposal to upgrade the bridge involving:

(a) a 40 mm concrete overlay,

(b) new expansion joints and deck drains, and

(c) painting of the structural steelwork.

MTC did a load test to determine the actual load capacity of the bridge before undertaking the proposal. This report gives the results of this test.

2/ BRIDGE DETAILS

Flack Bridge is a single span steel pony truss bridge, shown in Figure 1. The truss spans 21.3 m and has a 7.37 m wide concrete deck, see Figure 2. The 178 mm deep exposed concrete deck is laid over stringers (S12 X 31.8) at 1.22 m centres. The stringers are supported on floor beams (W27 X 94) fixed at each node on the bottom chord. Generally the truss consists of built up members of steel angles and/or plates and riveted connections. It has fixed bearings at the North end and expansion bearings on the South end.

The bridge investigation and evaluation study conducted by McCormick, Rankin and Associates [1] indicated the following:

1/ The concrete deck is in poor to fair condition, with significant scaling and spalling on the top 10 mm to 30 mm of the deck slab over more than 50% of the deck area. The top layer of reinforcement is exposed in a few locations. The concrete core strength is 74.6 MPa.

2/ The steel superstructure and steel bearings are in good condition but require removal of accumulated dirt and/or maintenance painting.

3/ The concrete sub-structure is in good condition.

4/ The truss evaluation shows that the critical structure component is the truss bottom chord at mid-span and therefore the bridge requires posting. The recommended posting level is given in Table 1.

3/ BRIDGE EVALUATION

The bridge superstructure was evaluated according to the Ontario Highway Bridge Design Code (OHBDC) [2]. The deck slab capacity determined through a simplified method given in Section 14 of the Code is approximately 750 kN for the existing 178 mm deep deck and 780 kN for the deck with 40 mm overlay. The deck resistance (Rn) required is approximately 392 kN. Consequently the deck has adequate capacity according to the OHBDC.

The stringer and floor beam forces calculated for dead and OHBDC level 3 live loads are given in Table 2a. For the purposes of this evaluation the concrete deck was assumed to be 225 mm throughout. The live load forces on the stringers were calculated using the GRIDS program [3]. The member forces and their resistances are compared in Table 2b. The stringers had adequate capacity while the floor beams had resistance values approximately 16.0% lower than required.

The truss member forces are given relative to member resistances in Table 3. The top chord showed a maximum of 23% and the bottom 14% under capacity. Most of the web members had adequate resistance values. The ultimate load capacity of the truss estimated using the IBUK program [3] was 1872 kN for the two truck loading case as opposed to the required capacity of 2205 kN, about 85% of that required under ultimate loads. Although the evaluation indicates that the floor beams and some truss members have lower capacity than that required to sustain OHBDC level 3 loads, it is likely that the tests would improve the estimated capacities since the bridge acting as a system can carry considerably higher loads.

4/ INSTRUMENTATION AND TEST LOADING

The proof load testing of a bridge structure usually involves loading the structure to induce member forces to reach the ultimate limit states defined by the OHBDC. At ultimate limit states the members can undergo considerable deformations and may be irreversibly damaged. To avoid such mishaps and to study the behaviour of various members under applied loads, required the structure to be heavily instrumented and closely monitored during the test. Both weldable and demountable type uniaxial strain gauges and deflection transducers were used (for locations see Figure 3). The stringers of the second panel and four floor beams were instrumented at their midspans. A single gauge was used to measure the strain at the bottom flange of each stringer and floor beam, since in past tests it was found that these values were adequate to establish the moment carried by the members. Critical truss members were instrumented with additional gauges to detect any premature failure. Vertical deflection transducers were attached to selected truss nodes and a floor beam. Transverse deflection transducers were used at both fixed and expansion bearing ends of the trusses.

The deck punching shear tests were performed at three locations with surfaces ranging from heavily spalled to little or no deterioration. The vehicular load during the test was applied by two MTC test vehicles described by Bakht and Csagoly [4]. The longitudinal and transverse positions of the trucks are shown in Figure 4. The two trucks were used either individually or together for truck loads of up to approximately 780 kN each. A single truck was used for loads of up to 890 kN. It was found that, considering the bridge location and volume of traffic, the two heavily loaded trucks with full dynamic load allowance passing the bridge simultaneously had a probability of 5.57×10^{-6} (see Bakht and Jaeger [5] and the Appendix). Therefore, two trucks each loaded to 890 kN were not used to load the bridge. It has been shown by Holowka [6], however, that based on longitudinal moment effects on simple span lengths of 24.4 m and 31.4 m the load effects of two 890 kN test trucks are equivalent within 1% of those specified in the OHBDC.

The strain gauges and deflection transducers were installed a week prior to the bridge testing. The weight of the vehicle was gradually increased and for each load level the trucks were placed either individually or together at predetermined locations (shown in Figure 4) to give critical force effects. The data for each loading were recorded using a computer-based data aquisition system. During the test, strains recorded were not unduly high and were found to be reversible.

5/ TEST RESULTS

The bridge structure carried the punching shear and truck loads applied without showing visible signs of distress, which in itself suggests that the bridge can carry the OHBDC live load without posting. The measurements made during the test, however, were later analysed closely to determine the structure's behaviour under the applied loads.

5.1/ Concrete Deck

The results of the punching shear test are illustrated in Figure 5. A maximum load of approximately 300 kN was applied and the deck behaved linearly, showing very little residual deformation. The ultimate load capacity required by the OHBDC is approximately 196 kN. However, the deck was found to be within elastic limits for the applied load of 300 kN. Since the punching shear tests were performed on deteriorated and undeteriorated slab, the test results confirm that deterioration has not affected deck capacity.

5.2/ Stringers

The stringer moments calculated using measured strains on the bottom flange and with composite or non-composite section properties are compared to the theoretical moments obtained using grid analysis in Figure 6. The theoretical and measured load distribution graphs are found to be similar except for single truck loading cases, where the theoretical moment graph shows a higher peak moment value than the measured moments. However, the measured moments obtained with composite section properties are closer to the theoretical moment graphs. Although the deck is of non-composite construction, the above observation confirms that the deck behaves partially composite under the applied loads. The evaluation reveals that the stringers have adequate capacity to carry OHBDC loads. The test shows the stringers are within the elastic limits under the applied test loads and this confirms that they are adequate to carry OHBDC loads without posting.

5.3/ Floor Beams

The theoretical and measured floor beam moments are compared in Table 4. The latter is obtained using the measured bottom flange strains and the floor beam section properties. Because of the relative rigidities of the floor system, applied loads on a floor beam generally get distributed among the adjacent floor beams. Bakht and Jaeger [7] have provided a methodology to account for this load distribution, and Table 4 contains the theoretical moments derived using these reduction factors. The theoretical and measured moment values compare well. The floor beams are found to be well within the elastic strain limits for steel under the applied truck loads. Using the estimated live load distribution factor of 0.6 for floor beams, the factored total moment is reduced from 1132.0 kNm to 761 kNm (see Table 2). The section moment capacity of 943.0 kNm is therefore adequate to carry the OHBDC loads without posting.

The measured and calculated deflections for the floor beam are given in Table 5. Although the theoretical deflections are generally greater than the measured deflections, the measured values are much closer to the calculated values.

5.4/ Truss Members

The truss tested is a determinate structure and since the failure of a member can prove to be disastrous to the bridge the truss was instrumented comprehensively. The stress variations observed on the individual components of the truss members are illustrated in Figure 7. For the end inclined chord, the angles showed higher stress than that of the plate element. However, for the top chord member the plate element showed higher stress than that of the angles. The stress variation for the web member was little or negligible while the bottom chord member showed considerable difference in stress between the inner and outer angle components.

For selected top chord, bottom chord and web members, the member forces calculated using elastic methods and the corresponding member forces obtained using the stress measurements are given in Table 6. In order to allow for the above stress variations within the components of a member,

the forces on individual elements were first calculated and added together to arrive at the measured member forces. The truck loading cases of a) two trucks on the bridge weighing 780.0 kN and b) a single truck on the bridge with total load of 890 kN were used to prepare Table 6. The measured top chord forces were generally lower than the theoretical member forces, the former ranging between 80 to 100% of the latter. Usually the actual structural system is stiffer than the theoretically idealized structure and this is the reason for the observed lower measured force values.

The measured values for the bottom chord members were below 30% of the calculated values and the chord adjacent to the bearing showed compressive forces. It was thought that the reduction may have been due to 1) the deck and stringers sharing the bottom forces and 2) the bearings not functioning adequately. The member forces were therefore calculated with the fixed bearings on both supports. The top chord member forces obviously were not affected but the bottom chord forces were. These values are also given in Table 6a. The bottom chord nearest to the bearings then carries a compressive force, which explains the compressive forces observed on this member for some load cases. For the bottom chord 4-9, there is better agreement between the measured and calculated tensile forces. Because the bottom chords are incapable of sustaining high compressive force, the kind of force reduction observed on member 4-9 is not practical. Therefore, without repeating the test to isolate the effects of frozen bearings, it is concluded that the force reduction observed on the bottom chord is due to 1) deck and stringers sharing the bottom chord forces and 2) the free bearings not functioning adequately.

Although for some web members, the measured values were greater than the theoretical values, the web members usually showed lower measured force values than the corresponding calculated values. All the truss members were within the elastic stress limit under the applied loads and this confirms that the truss can sustain OHBDC loads without posting.

The measured and calculated movements of the truss with two trucks each loaded to 780 kN is given in Table 7. The measured deflection values are lower than the calculated ones and range between 30 to 60% of the theoretical values. Although the deflection values obtained with fixed bearing at both supports are greater than the measured values, the measured values are closer to the calculated values, see Table 7. The horizontal movements observed at the free and fixed bearings were quite low and suggest that the bearings could have been frozen.

The pony truss top chords are laterally restrained by the inverted portal arrangement provided by the floor beam and vertical ties. The increased lateral movement could substantially reduce the ultimate load capacity of the truss. The lateral movements of the top chord for various loading cases were observed during the load test and Figure 7 illustrates them. The lateral movement was unaffected by the applied load levels. The measured movements appear very low, indicating that the truss was not distressed by the heavy truck loads.

6/ CONCLUSIONS AND RECOMMENDATIONS

The bridge evaluation investigations performed as per OHBDC for level 3 live loads indicate the following:

- The slab has adequate capacity.
- The stringer has excess capacity while the floor beams have approximately 16.0% less strength than required.
- For pony trusses, the top chord members show a maximum of 23% under capacity and the bottom chord shows 14% under capacity. Most of the web members have adequate resistance values.

The bridge test and post test analysis indicate the following:

- The slab is within the elastic load limit under an applied load of 300 kN and meets and surpasses the ultimate load capacity of 196 kN required by the OHBDC.
- The theoretical stringer moments are found to be very close to the measured moments obtained with composite section properties. Although the deck is constructed non-compositely, the test results indicate that the stringer functions compositely. The stringers behave elastically under the applied loads, confirming the evaluation study results that the stringers have adequate capacity to carry OHBDC loads.
- The theoretical floor beam moments which were reduced to account for the load distribution between adjacent beams compare well with the measured floor beam moment values. Using the same reduction factors, the factored moment due to OHBDC level three loads is 761 kNm, well below the section capacity of 943.0 kNm. The floor beam is well within elastic strain limits for structural steel under the applied truck loads.

- For the truss, the measured top chord member forces are generally lower than the theoretical member forces, the measured values ranging between 80 to 100% of the theoretical values. The bottom chords give much lower measured values than the corresponding theoretical member forces, the former usually being below 30% of the latter. An analysis with fixed bearings reduces the difference between the measured and calculated force values. However, it is concluded that the force reduction observed on the bottom chord is due to a) deck stringers sharing the bottom chord forces and b) the effect of the frozen bearing. Although for some web members the measured values are greater than the theoretical values, most of them show lower measured force values than those obtained by theoretical analysis. All truss members are within the elastic limits under applied test loadings which confirms that the truss has adequate capacity to carry OHBDC loads.

- The truss deflections range between 30 and 60% of the theoretical values. The measured deflection values are, however, found to be closer to the calculated values with fixed bearings at both supports. The lateral movement of the truss remained constant under two truck loading cases with various load levels. The free bearings movement monitored suggests that they may not be functioning adequately.

As result of the bridge tests and analysis, the following recommendations were made:

- The bridge does not require posting.

- The deck requires an immediate overlay to control scaling, spalling and exposure of reinforcement. The bearings of the pony trusses require immediate maintenance.

- New expansion joints, deck drains and structural steel painting are necessary.

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APPENDIX

Probability of truck weight exceeding 700 kN in Ontario, using Csagoly and Knobel [8] $= 2.3 \times 10^{-4}$

Assumed truck traffic per day $= 50$

Number of trucks passing the bridge per day $50 \times 2.3 \times 10^{-4}$ $= 0.0115$

Assumed time interval for which the truck causes the worst effects $= 1$ s

Probability of a truck exceeding 700 kN being on the bridge in a particular 1 s interval in a day $\dfrac{0.0115}{24 \times 60 \times 60}$ $= 1.33 \times 10^{-7}$

Probability of two trucks each exceeding 700 kN being on the bridge in the same 1 s interval in a day $(1.33 \times 10^{-7})^2$ $= 1.77 \times 10^{-14}$

Number of 1 s intervals in the next 10 year period $10 \times 365 \times 24 \times 60 \times 60$ $= 3.15 \times 10^{8}$

Total number of times two trucks each exceeding 700 kN are on the bridge in the same 1 s interval in the next 10 years $1.77 \times 15^{-14} \times 3.15 \times 10^{8}$ $= 5.57 \times 10^{-6}$

Table 1/ Recommended Postings

LOADING	SCALE DOWN FACTOR AND POSTING			
	Existing Condition		With 40 mm Overlay	
	S.D.F.	Posting, Tonne	S.D.F.	Posting, Tonne
OHBDC Level 1	0.95	24	0.87	22
Level 2	0.76	30	0.70	26
Level 3	0.76	37	0.70	33

S.D.F. - Scale Down Factor

Table 2a/ Unfactored Forces on Stringers and Floor Beams

MEMBER	MEMBER CRITICAL MOMENTS, kNm	
	Dead Load	Live Load
Stringer	11.2	40.1
Floor Beam	170.3	509.7

Table 2b/ Factored Forces and Resistances on Stringers and Floor Beams

MEMBER	FACTORED MEMBER MOMENTS, kNm	FACTORED MEMBER RESISTANCES, kNm	
		Laterally Supported	Laterally Unsupported
Stringer	86.4	142.2	122.0
Floor Beam	1132.0	943.0	--

Table 3/ Truss Member Forces Related to Member Resistance

MEMBER	FACTORED MEMBER FORCES, kN			Member Resistances
	Dead Load	Live Load	Total	
Top Chord				
1-2	-465.6	-928.2	-1393.8	1073.0
2-6	-561.6	-1088.6	-1650.2	1352.0
Bottom Chord				
1-4	+351.6	+709.8	+1061.4	1068.0
4-9	+630.0	+1177.3	+1807.3	1553.0
Web				
2-3	+116.4	+395.8	+512.2	1385.0
2-4	+273.6	+617.6	+891.2	813.0
4-6	-90.0	-368.6	-458.6	-576.0

Table 4/ Calculated Floor Beam Moments Related to Measured Floor Beam Moments for Various Loading Cases

LOAD CASE	FLOOR BEAM MIDSPAN MOMENTS, kNm		
	Calculated	With Correction for Load Sharing	Measured
Floor Beam 1			
Level 4/Line 1/Pos 1	360.0	216.0	167.9
Level 4/Line 2/Pos 2	363.9	218.3	228.3
Level 4/line 3/Pos 1	723.9	434.3	377.7
Level 5/Line 1/Pos 1	425.5	255.3	198.6
Level 5/Line 2/Pos 1	425.5	255.3	258.1
Floor Beam 2			
Level 4/Line 1/Pos 3	360.0	216.0	165.4
Level 4/Line 2/Pos 3	363.9	218.3	177.4
Level 4/Line 3/Pos 3	723.9	434.3	335.8
Level 5/Line 1/Pos 3	425.5	255.3	199.7
Level 5/Line 2/Pos 3	425.5	255.3	198.6

Table 5/ Comparison of Measured and Calculated Floor Beam Deflections

LOAD CASE	CALCULATED MIDSPAN DEFLECTIONS, mm	MEASURED MIDSPAN DEFLECTIONS, mm
Floor Beam 3		
Level 4/Line 1/Pos 4	2.6	2.0
Level 4/Line 2/Pos 4	2.8	2.2
Level 4/Line 3/Pos 4	5.3	4.0
Level 5/Line 1/Pos 4	3.1	2.4
Level 5/Line 2/Pos 4	3.1	2.4

Table 6a/ Member Forces on the East Truss due to Truck with 780.0 kN Load

	LINE 1								LINE 3							
Member	Pos 1		2		3		4		Pos 1		2		3		4	
	Cal.	Meas.	Cal.	Meas.	Cal.	Meas.	Calc.	Meas.	Cal.	Meas.	Cal.	Meas.	Cal.	Meas.	Cal.	Meas.
Top Chord																
1-2	-501.8	451.3	-483.7	445.5	-410.0	422.8	-340.5	339.9	-726.2	664.8	-703.8	686.3	-599.1	628.2	497.4	541.8
2-6	-521.2	458.4	-586.6	509.6	-559.1	504.7	-530.9	440.1	-756.8	692.5	-850.1	762.4	-870.6	764.9	772.7	699.5
Bottom Chord																
1-4	387.8	14.9	375.3	-12.2	319.7	-27.5	265.5	-66.7	488.2	45.5	544.6	22.0	462.3	-9.2	386.4	-55.0
									(-6.0)		(-74.9)		(-142.7)		(-208.0)	
4-9	585.2	119.0	624.1	130.7	628.3	129.9	633.8	145.9	839.4	211.3	903.1	241.0	912.3	260.7	921.5	274.8
									(316.2)		(327.0)		(345.7)		(349.0)	
Web																
2-3	207.1	191.2	136.2	121.9	33.4	88.7	0.0	48.9	300.7	275.8	196.9	219.0	48.9	123.1	0.0	25.2
2-4	179.3	202.8	272.4	288.4	362.6	310.9	346.1	302.1	258.6	301.6	396.4	392.4	527.1	464.3	501.1	453.8
4-6	-69.5	-37.9	-41.7	37.9	-33.4	43.2	-125.1	99.7	-91.2	44.9	-60.3	57.8	48.3	71.4	-182.1	128.3

( ) — force calculated with fixed bearings.

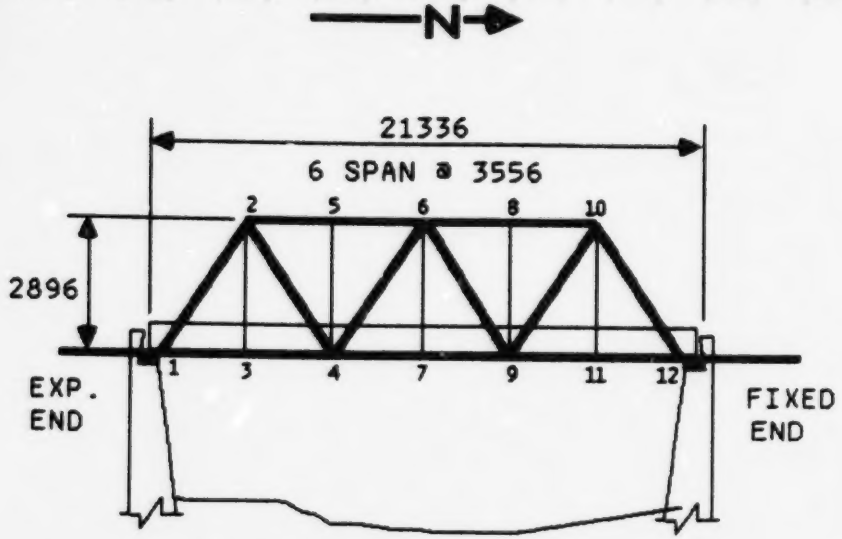
Table 6b/ Member Forces on the East Truss due to Two Trucks Loaded to 890.0 kN Load

LINE 1								
MEMBER	Pos 1		2		3		4	
	Cal.	Meas.	Cal.	Meas.	Cal.	Meas.	Cal.	Meas.
Top Chord								
1-2	576.8	524.0	556.0	525.2	479.6	472.3	397.5	408.9
2-6	631.1	526.3	667.2	576.6	681.1	578.9	612.9	526.0
Bottom Chord								
1-4	444.8	27.4	430.9	13.4	368.4	-10.4	307.2	-9.8
4-9	670.0	159.5	715.9	179.2	722.8	189.1	728.4	197.1
Web								
2-3	239.1	217.8	158.5	164.1	40.3	92.6	0.0	28.7
2-4	204.3	103.6	312.8	130.9	417.0	144.3	386.4	142.0
4-6	-80.6	9.7	-45.9	20.2	-36.0	32.6	-139.0	51.1

Table 7/ Measured and Calculated Movements of Truss Under Two Trucks Loaded 780.0 kN

DISPLACEMENT GAUGE NO.	LINE 3							
	Pos 1		2		3		4	
	Cal. mm	Meas. mm	Cal. mm	Meas. mm	Cal. mm	Meas. mm	Cal. mm	Meas. mm
Vertical Displacement Transducers								
56	13.1	6.6	13.6	6.8	12.5	6.1	11.2	5.4
57	20.1	6.8	22.5	13.3	23.1	13.5	21.3	12.6
58	23.3	13.1	25.4	14.8	26.0	15.4	25.9	15.2
59	22.7	12.4	25.2	13.8	25.9	14.3	25.8	14.1
Horizontal Displacement Transducers								
52	8.2	0.36	8.9	0.38	8.9	0.38	8.7	0.36
53	0.0	0.0	0.0	0.17	0.0	0.38	0.0	0.40
54	8.2	0.38	8.9	0.41	8.9	0.41	8.7	0.35
55	0.0	0.05	0.0	0.22	0.0	0.45	0.0	0.62

Figure 1/ Flack Bridge



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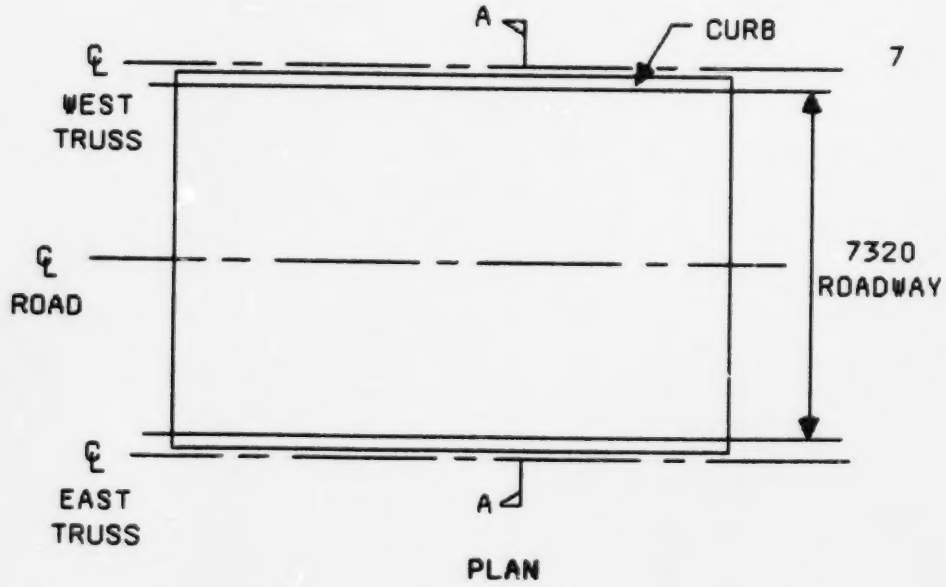
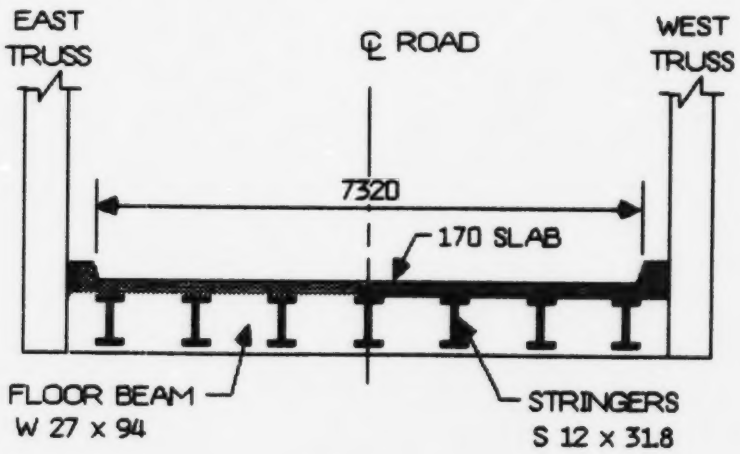


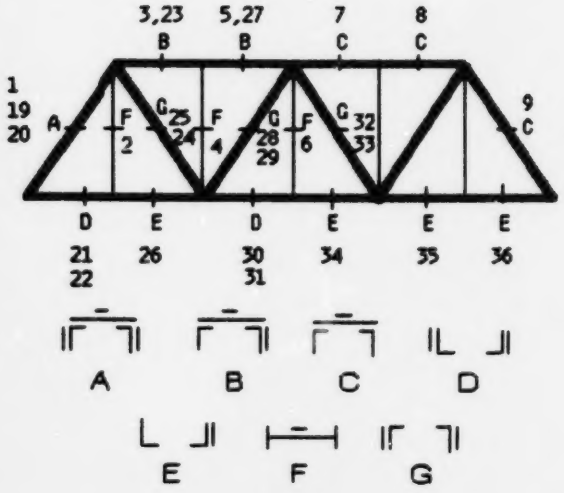
PLAN



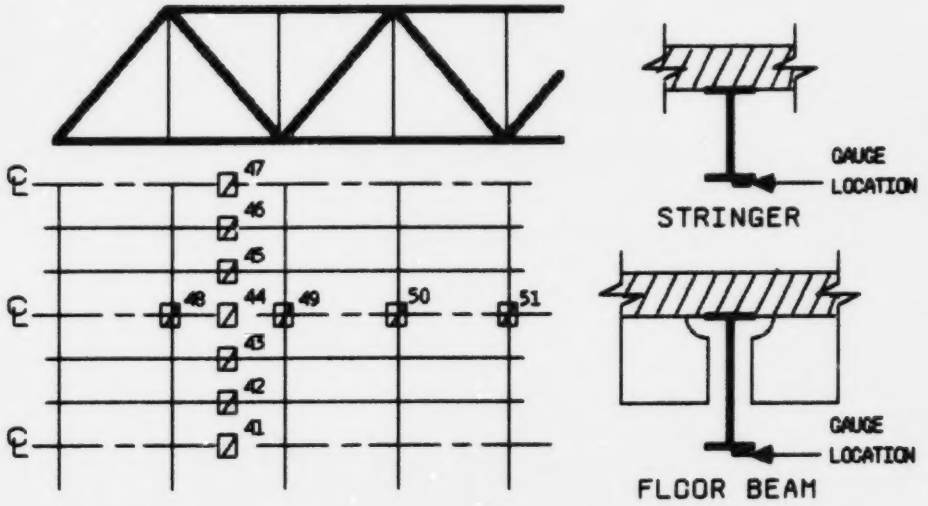
SECTION A-A

MEMBER	SECTION USED
1 - 2	2L^{S} 6 x 3 1/2 x 1/2 1 No.5 10 x 3/8 PLATE
2 - 6	2L^{S} 6 x 4 x 1/2 1 No.5 10 x 3/8 PLATE
1 - 4	2L^{S} 6 x 4 x 1/2
4 - 9	2L^{S} 6 x 4 x 3/4
2 - 4	2L^{S} 6 x 3 1/2 x 3/8 TIED WITH PLATE
4 - 6	2L^{S} 6 x 3 1/2 x 3/8, LACED
2 - 3	WF 10 x 8 x 16 @ 39

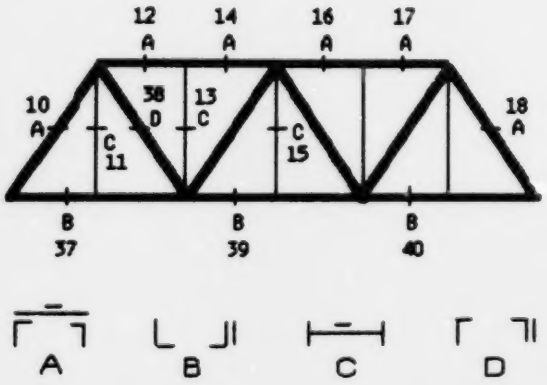
Figure 2/ Bridge Details



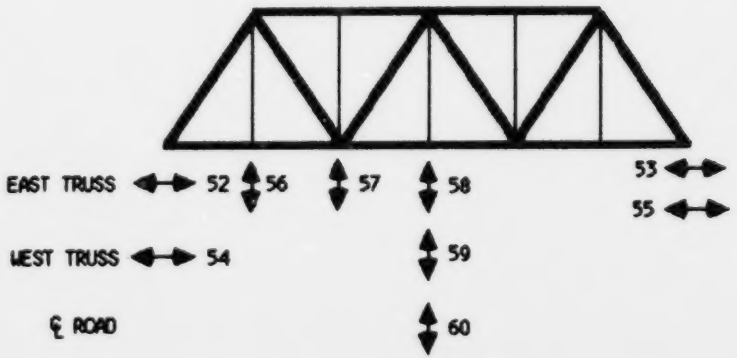
EAST TRUSS INSTRUMENTATION



STRINGER, FLOOR BEAM INSTRUMENTATION

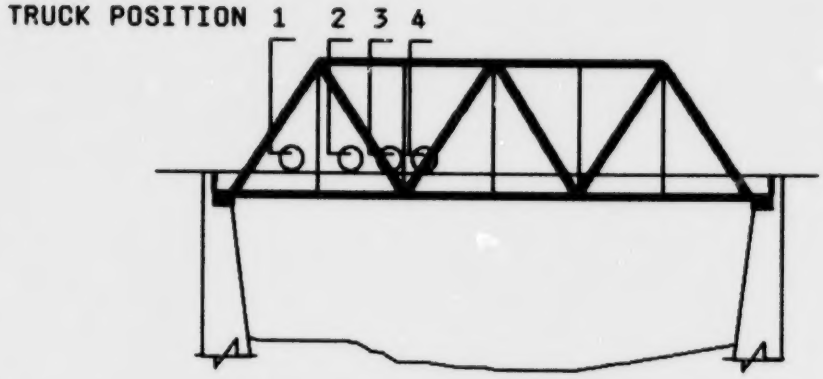


WEST TRUSS INSTRUMENTATION

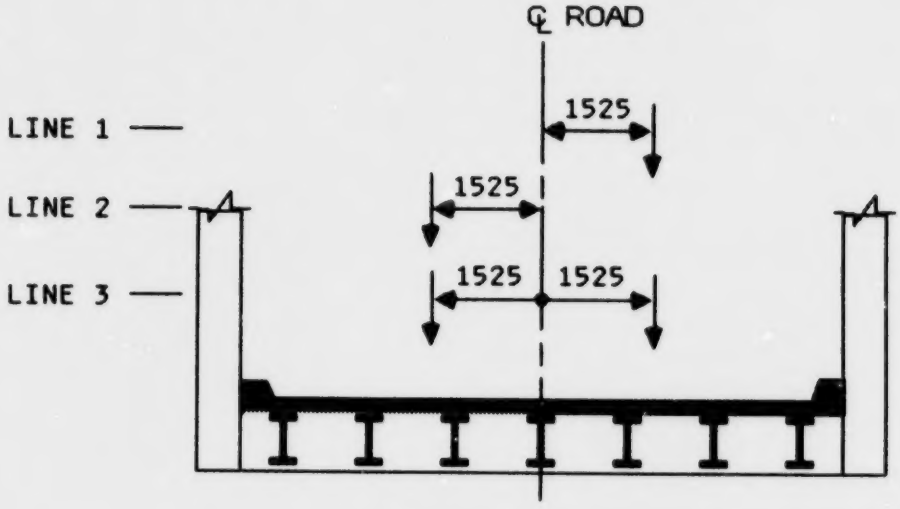


DEFLECTION TRANSDUCER POSITIONS

Figure 3/ Instrumentation of Bridge



LONGITUDINAL POSITIONS OF THE TRUCK



TRANSVERSE POSITION OF THE TRUCK

AXLE WEIGHTS OF TEST VEHICLES

	LOAD LIFT No.	WEIGHT ON AXLE No., kN 1	2	3	4	5	TOTAL TRUCK WEIGHT, kN
"KENWORTH"	24 BLKS.	47.7	92.7	92.7	105.2	105.2	443.5
	36 BLKS.	49.4	115.5	115.5	136.9	136.9	554.2
	48 BLKS.	51.7	138.7	138.7	168.9	165.9	666.9
	60 BLKS.	53.5	161.9	161.9	201.1	201.1	779.5
	72 BLKS.	55.3	180.5	180.5	237.7	237.7	891.7
"MAC"	24 BLKS.	43.6	90.1	90.1	104.7	104.7	433.2
	36 BLKS.	44.6	113.2	113.2	137.7	137.7	546.4
	48 BLKS.	45.9	136.0	136.0	170.3	170.3	658.5
	60 BLKS.	46.8	159.2	159.2	203.3	203.3	779.5

Figure 4/ Truck Positions and Loading

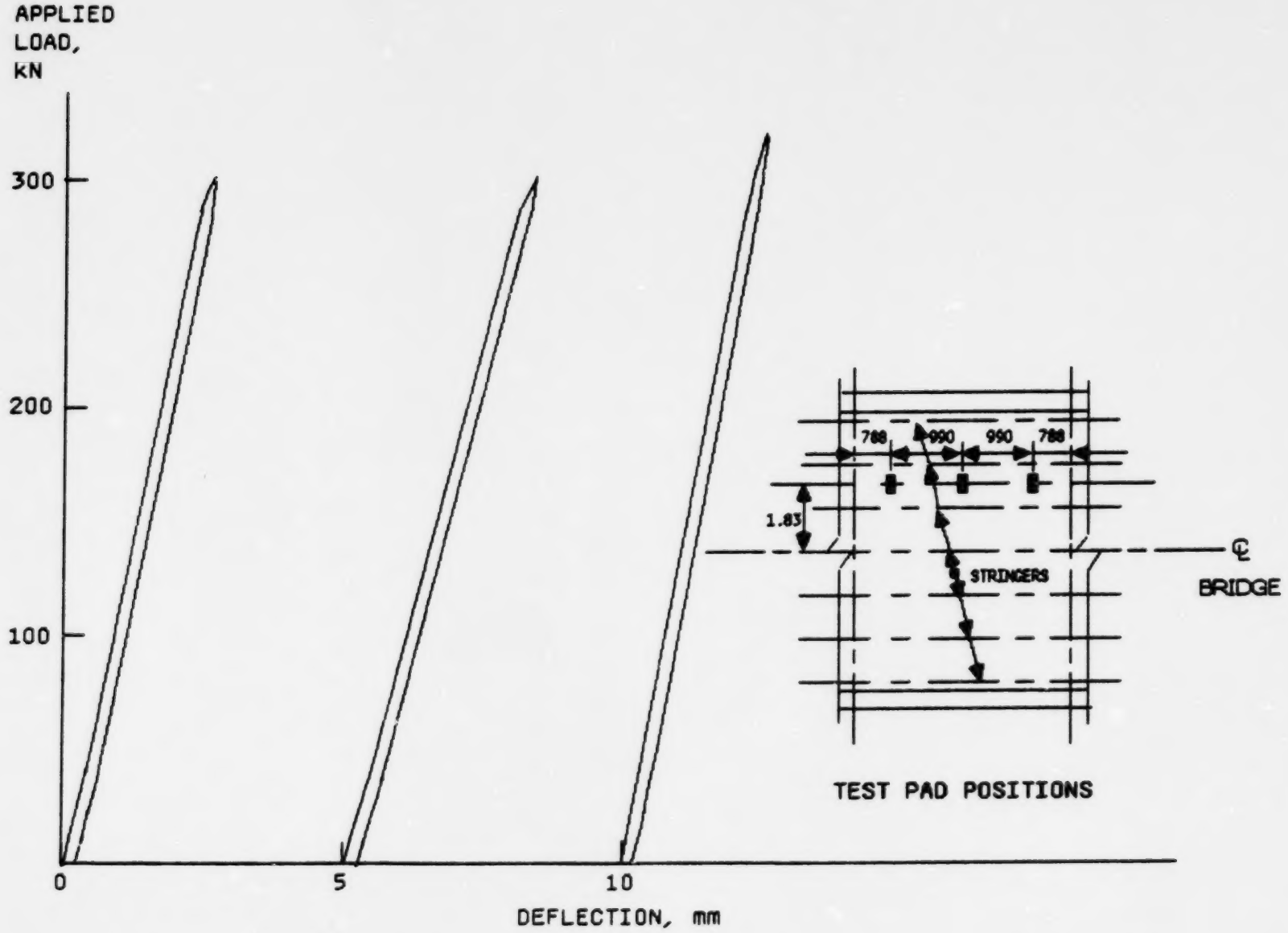


Figure 5/ Deck Behaviour Under Punching Shear Load

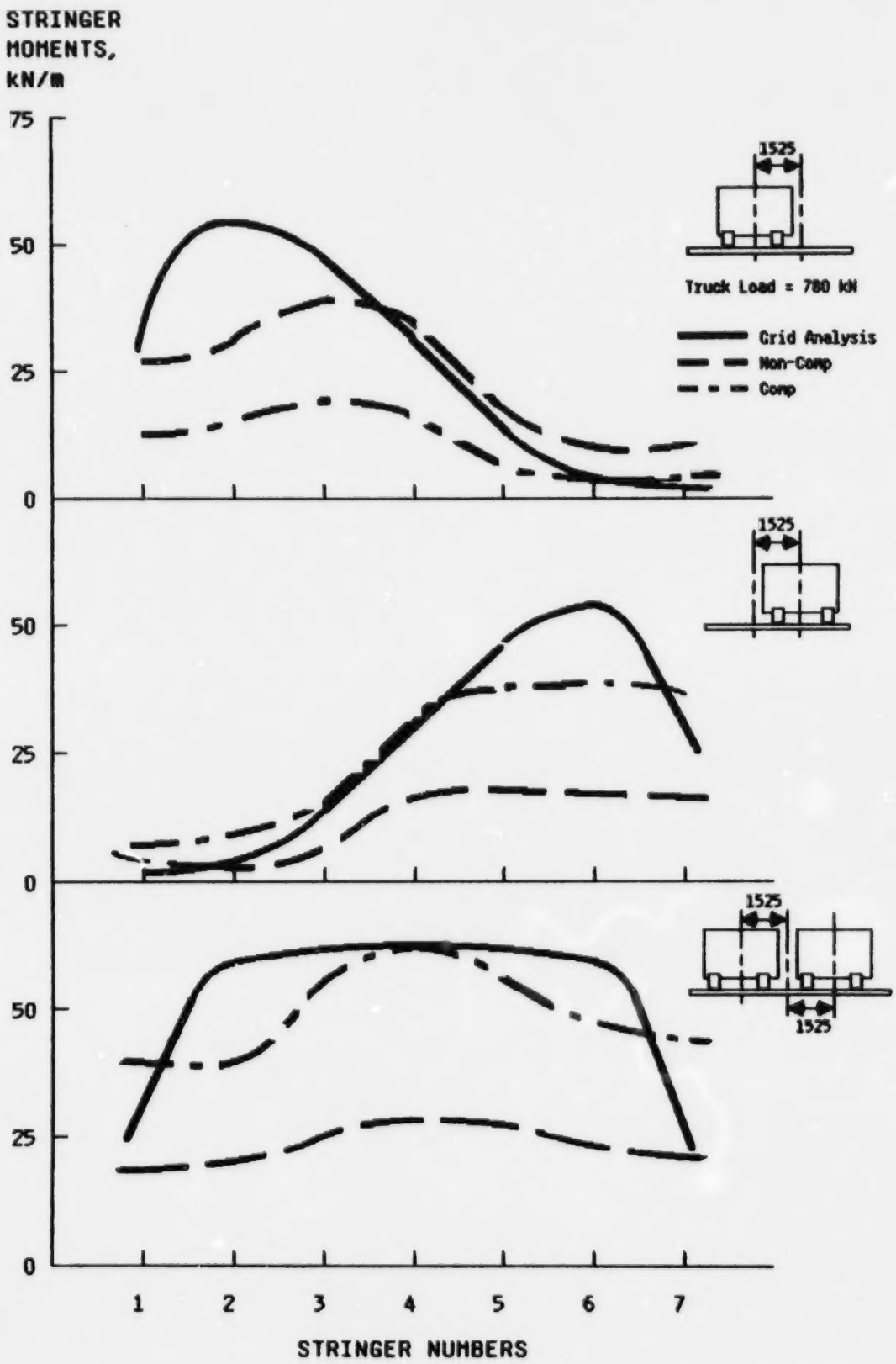


Figure 6a/ Theoretical Stringer Moments Related to Measured Stringer Moments

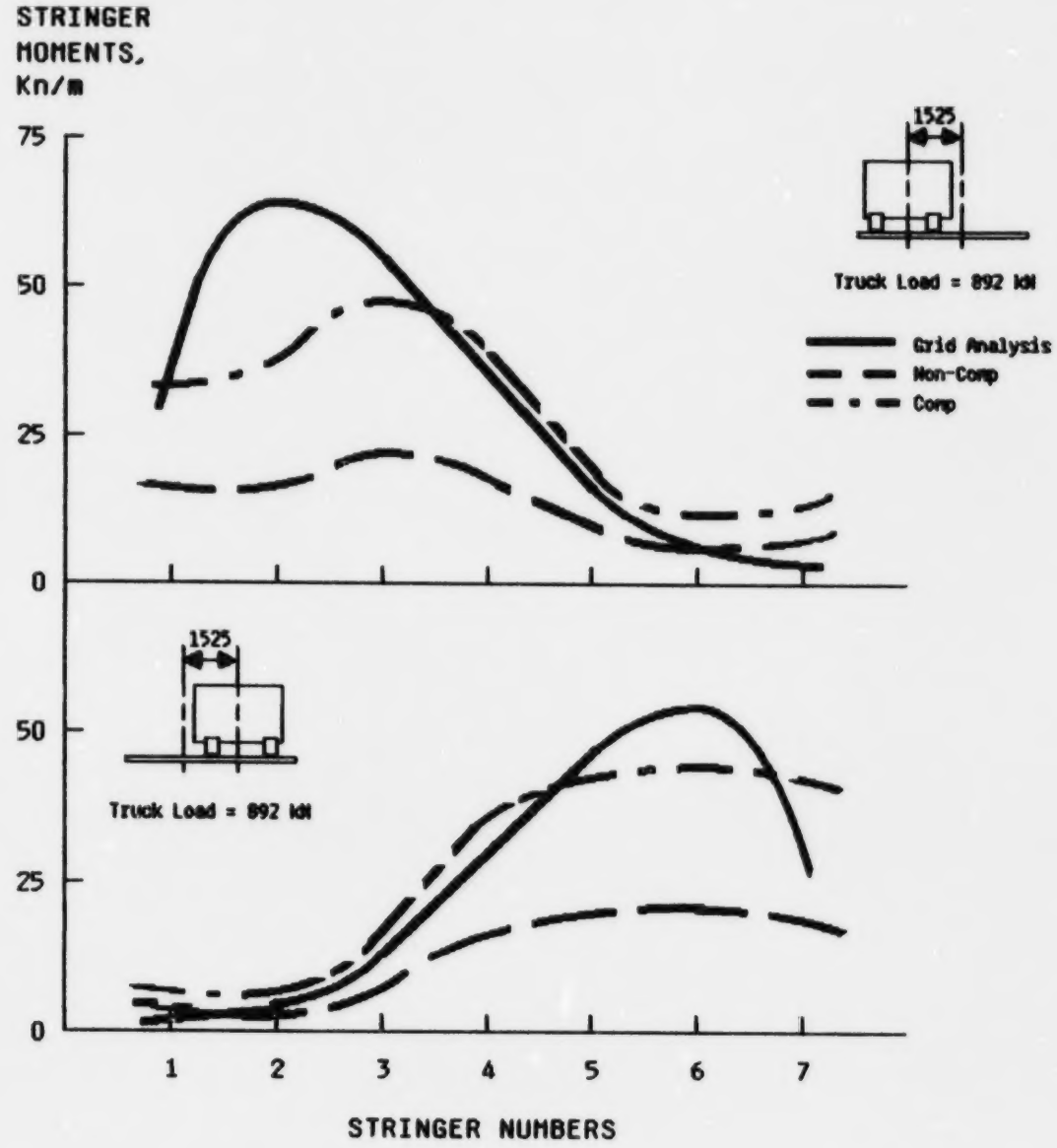


Figure 6b/ Theoretical Stringer Moments Related to Measured Stringer Moments

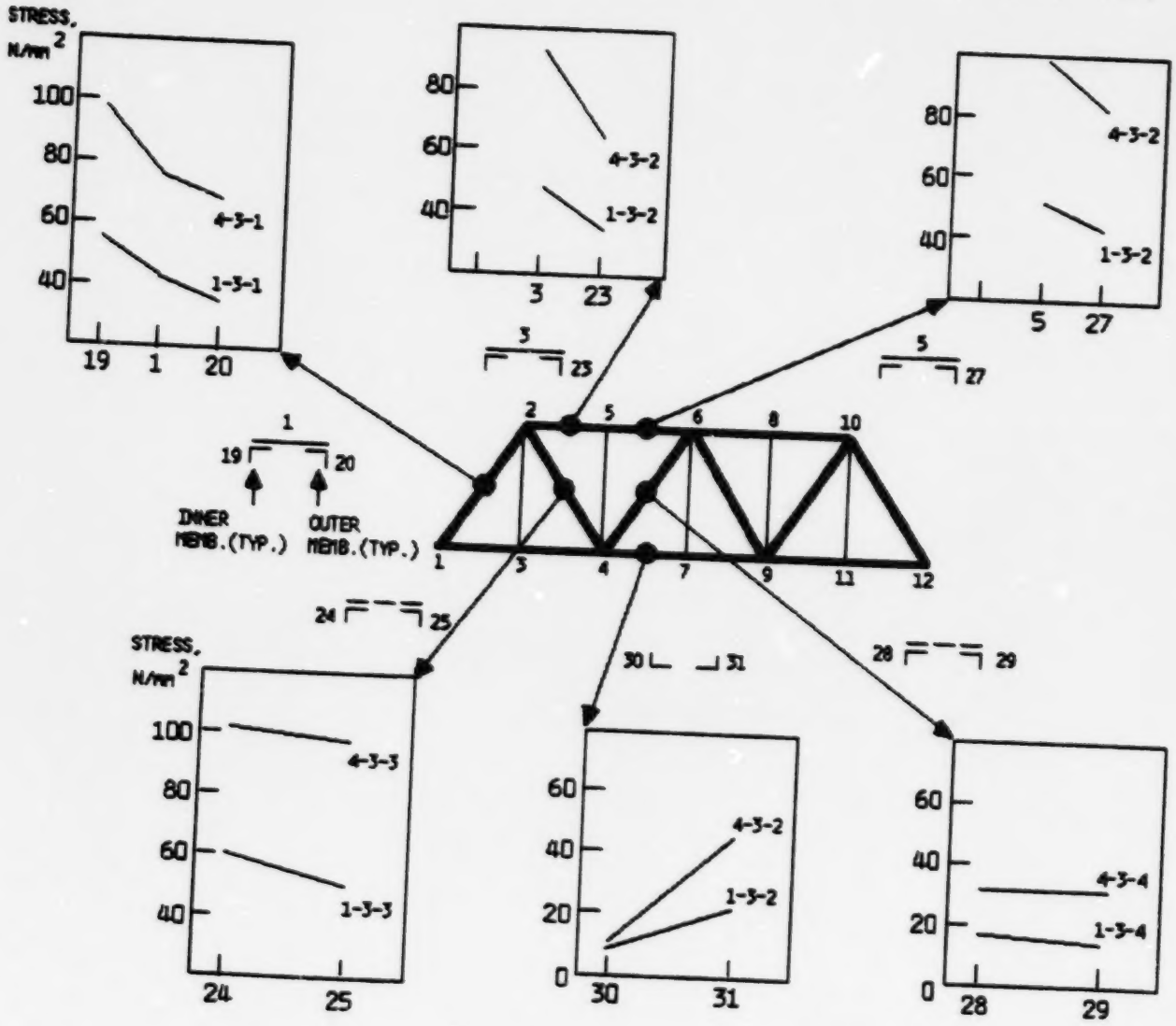
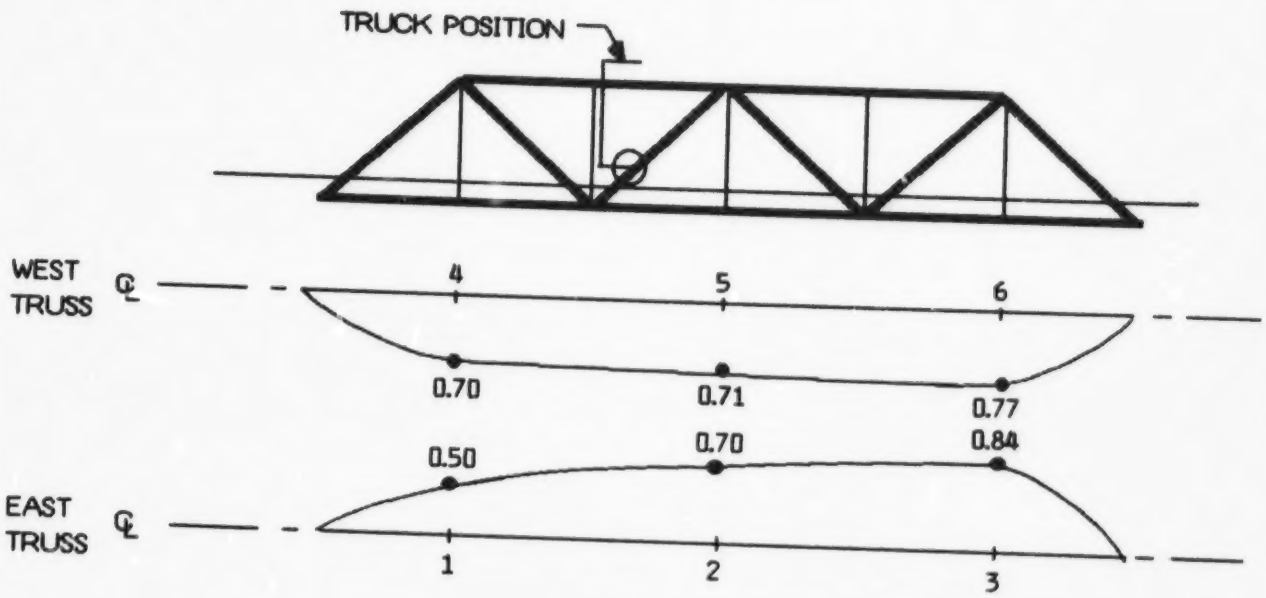


Figure 7/ Typical Stress Variations Observed in the Components of the Built Up Tress Members



TOP CHORD LATERAL DEFORMATION

TABLE LATERAL DEFORMATION OF TRUSSES UNDER LINE 3 LOADS

POSITION	LATERAL DEFORMATION, mm			
LEVEL	1	2	3	4
1	0.500	0.500	0.500	0.500
2	0.699	0.699	0.700	0.700
3	0.837	0.837	0.838	0.838
4	0.700	0.700	0.700	0.700
5	0.713	0.713	0.713	0.713
6	0.772	0.772	0.772	0.772

Figure 8/ Lateral Movement of the Truss